SEISMIC DESIGN AND TESTING OF A 5-STORY PRECAST CONCRETE BUILDING WITH DUCTILE CONNECTIONS

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KEYNOTE ADDRESS

In the final phase of the PRESSS (**Pre**cast **S**eismic **S**tructural **S**ystems) program, a large-scale five-story precast concrete building is tested under simulated seismic loading. A brief summary of various structural features incorporated in the PRESSS building and test plans are presented in this paper. Construction and test preparations of this precast building have been already completed at UCSD Charles Lee Powell Structural Laboratory. All seismic testing of the building is scheduled to be completed by September 1999.

1. INTRODUCTION

The PRESSS program has been on going for the past 10 years with an overall objective of developing seismic design recommendations for precast concrete systems[1,2]. In the initial two phases of this program, experimental and analytical studies of ductile connection precast elements for frame and wall structures, as distinct from strong-connection precast structures which attempt to emulate monolithic reinforced concrete construction, were conducted. In the final phase, a large-scale precast five-story building utilizing different connection details is tested under simulated seismic loads[2].

2. TEST BUILDING

Based on five-story prototype buildings with 100 x 200 sq. ft (30.5x61.0 m^2) in plan (per floor), 12 ft 6 in. (3.8 m) story height and 25 ft (7.6 m) bay length, dimensions of the test building were established. It was first determined that, for seismic testing purposes, it would be only necessary to model 50 x 50 sq. ft (15.75x15.75 m^2) plan area of the prototype buildings with 2 bays in each direction, provided the additional tributary mass of section of floor supported by gravity frames was modelled in the test control. The test building was then modeled at 60% scale of the resized prototype buildings in order to accommodate it inside the Charles Lee Powell Structural Laboratory at the University of California at San Diego (UCSD). This resulted in the test building having 30 x 30 sq. ft (9.15 x 9.15 m^2) in plan, 7 ft 6 in. (2.3 m) story height and 15 ft (4.6 m) bay length and modeling all critical connections of a real building.

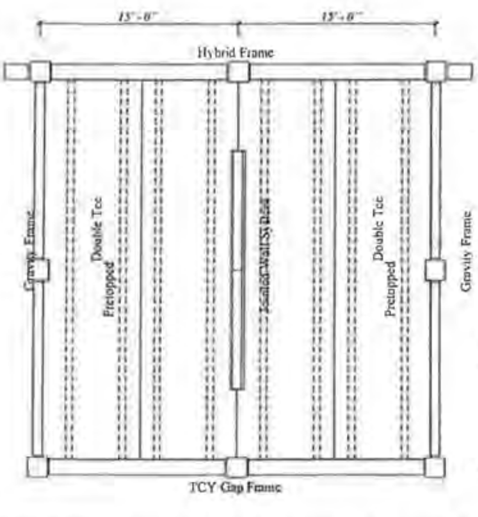

Fig. 1 Floor plan of the test building at Levels 1 - 3.

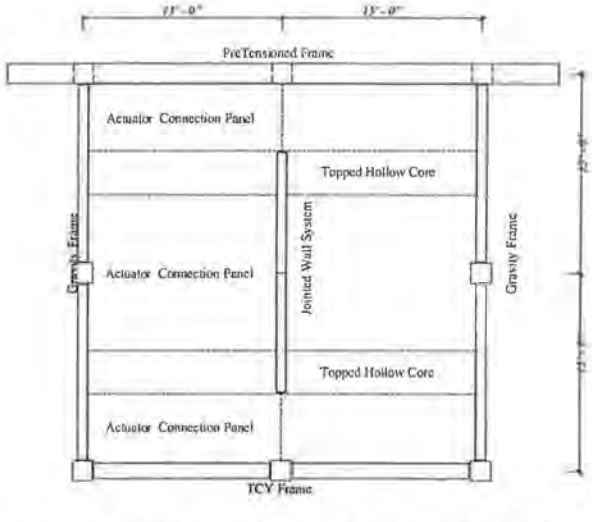

Fig. 2 Floor plan of the test building at Levels 4 & 5.

In one direction of the PRESSS building, seismic resistance is provided by precast frame systems, with a precast wall system and gravity frames in the orthogonal direction (Figs. 1 and 2). Four different beam-to-column connection details, based on the past PRESSS research program[3,4], are modeled at different levels in the two parallel seismic frames. The prestressed frame shown in Fig. 3 models the hybrid and pretensioned connections while the TCY (Tension-Compression Yielding) frame in

Fig. 4 models the TCY gap and TCY connections. In each frame, the connection type is identical in the first three floors and a different connection is used in the fourth and fifth floor levels.

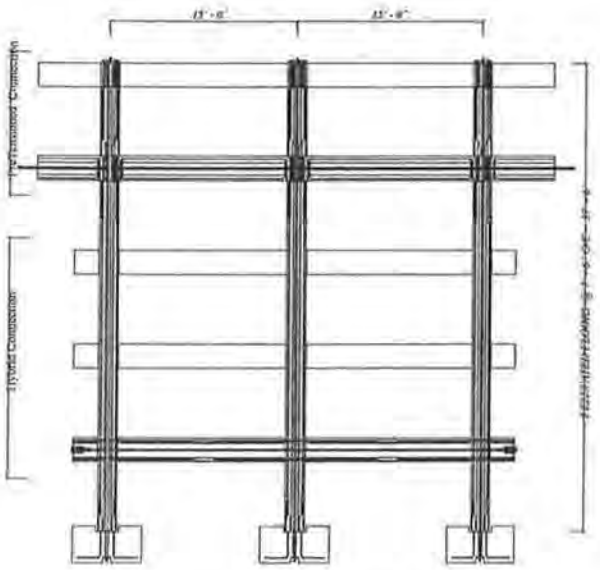

Fig. 3 Elevation of Prestressed seismic frame.

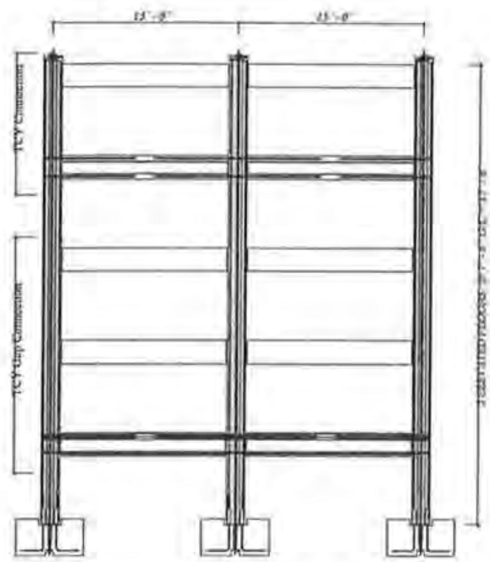

Fig. 4 Elevation of TCY seismic frame.

A brief description of each of the frame connections is as follows:

1. Hybrid frame connection (Fig. 5) – Beam-to-column frame connection is established with unbonded post-tensioning through the centre of joint and field placement of mild steel reinforcement in ducts across the joint interface closer to the top and bottom beam surfaces. These ducts are grouted to ensure adequate bond for the reinforcement prior to post-tensioning.

2. PreTensioned frame connection (Fig. 6) – Continuous partially bonded pretensioned beams are connected to column segments extending from the top of beam at one floor level to the bottom of beam at the level above. The moment connection between the beam and column is established by extending the column mild steel reinforcement below the beam through sleeves located in the joint. The extended reinforcement is spliced to the column longitudinal reinforcement at the next level adjacent to the joint.

3. TCY gap connection (Fig. 7) – Mild steel reinforcing bars placed in grouted sleeves at the top of the beam and unbonded post-tensioning at the bottom of the beam provide the necessary moment resistance at beam ends. Beams and columns are separated by a small gap to avoid elongation of the beam due to seismic action. This gap is partially grouted at the interface over 6" at the bottom of the beam with the post-tensioning force acting at the centre of grout.

4. TCY connection (Fig. 8) - Behaviour of monolithic reinforced concrete connections is emulated in this connection with top and bottom mild steel reinforcement in grouted sleeves across the beam-to-column interface.

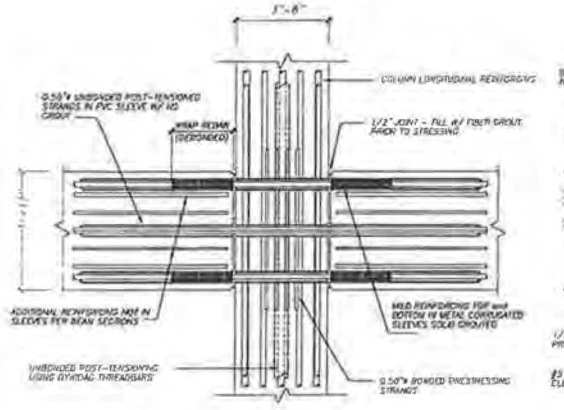

Fig. 5 Hybrid frame connection.

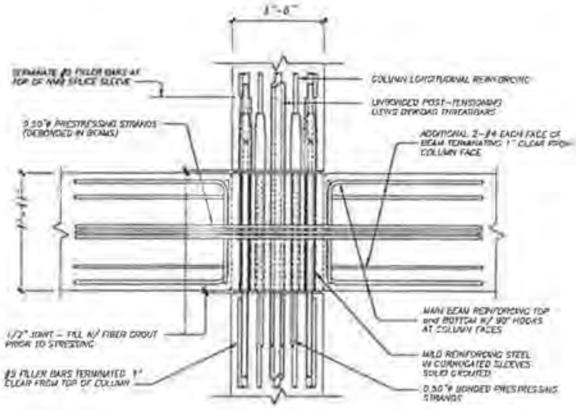

Fig. 6 PreTensioned frame connection.

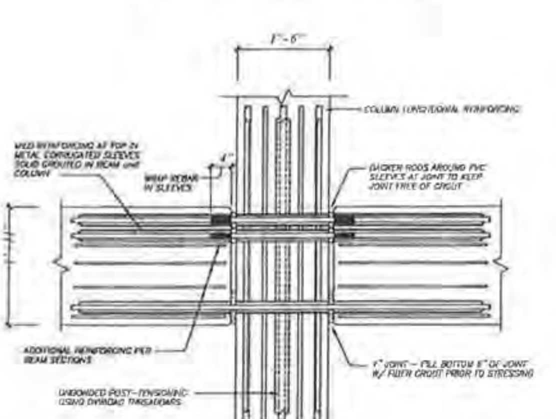

Fig. 7 TCY gap frame connection.

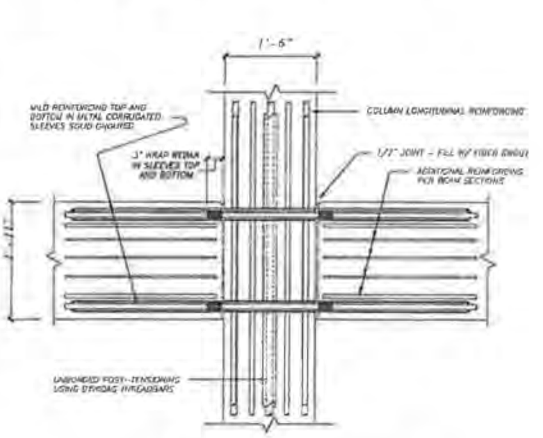

Fig. 8 TCY frame connection.

The walls in the orthogonal direction contain unbonded vertical prestressing, with special energy dissipating connectors located in a vertical construction joint between wall elements (see Fig. 9a). U-shaped stainless steel flexural plates (UFP), as shown in Fig. 9b, are used as connectors based on an earlier phase of the PRESSS investigation[5]. Two precast flooring systems are also included in the test building. The first three floors are constructed using pretopped double tees (Fig. 1) while hollow-core panels are used in the upper floors (Fig. 2), with an in-situ topping.

Combinations of two building systems and four ductile frame connections adopted in the PRESSS test building effectively provide experimental verifications of seismic behavior of five different precast prototype buildings. In addition, application of the two most popular precast flooring systems to different seismic resistant building systems is also examined.

3. DESIGN PROCEDURE

The PRESSS building was designed using the direct-displacement based approach (DBD)[6] to sustain a maximum drift of 2% under a design level earthquake that represents the 1997 UBC Zone 4, Soil type S_c acceleration spectrum[7].

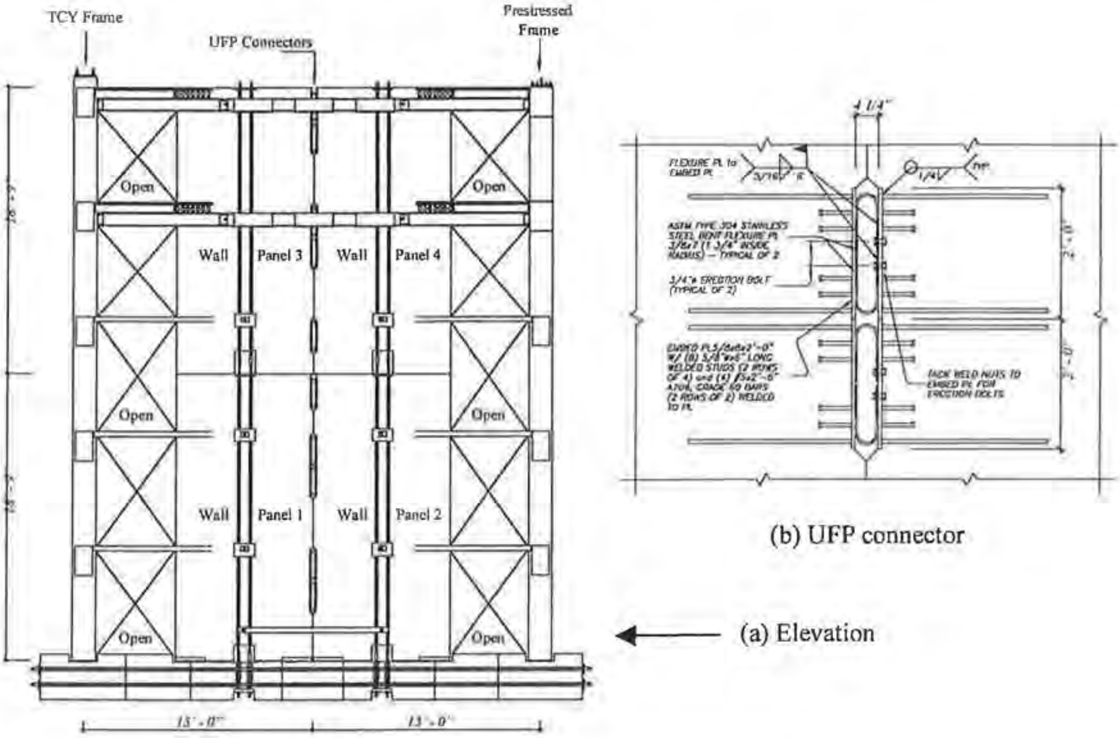

Fig. 9 Jointed precast wall system.

The UBC provisions do not include design displacement spectra that are required in the DBD approach. Hence, the 5% damped displacement response spectra included in Appendix G of the draft SEAOC Bluebook[8] were used in the PRESSS building design. As detailed later in this paper, acceleration design spectra in UBC and SEAOC Bluebook are comparable, and thus utilizing SEAOC displacement spectra in the design of the test building was considered satisfactory.

The reason for using the DBD approach to designing the PRESSS building was that force based design does not sufficiently account for behavior of jointed precast systems. Furthermore, the R factors given in design codes as part of the force based design are also intended for precast systems emulating monolithic concrete connections, rather than for some of the connections incorporated in the PRESSS building. As a result of applying the DBD procedure, less conservative design base shears were obtained in both the frame and wall directions of the test building when compared to those obtained from the force based design method[2]. Reduced base shear results in significant cost savings in addition to improving performance over traditional precast systems.

4. SEISMIC TEST PLAN

Seismic testing of the PRESSS building will be independently conducted in the two orthogonal directions. In each direction, three different test schemes, namely the stiffness measurement test, pseudodynamic test and inverse triangular load test, will be used.

The stiffness measurement test is a quasi-static loading test through which the stiffness matrix of the test building is formulated. The stiffness matrix, which is updated following increased intensity of the lateral seismic loading, is useful in (a) determining the appropriate integration time step when explicit schemes are used in solving the equation of motion in the pseudodynamic testing procedure, (b) improving convergence of implicit integration schemes in the pseudodynamic testing procedure, (c) characterizing structural behavior consistent with the direct-displacement based approach, and (d) monitoring damage levels using stiffness as a damage indicator.

Significant portions of the testing in the two orthogonal directions will be performed using a pseudodynamic testing procedure. In this procedure, the external dynamic load is applied to the structure quasi-statically through ten on-line controlled hydraulic actuators. Combining numerical computation and experimental measurements, the pseudodynamic test is carried out using the concept outlined in Fig. 10. To ensure speedy convergence of solutions and minimize error propagation in this test procedure, algorithms developed as part of the TCCMAR masonry building test(9) are used with some modifications. Several segments of earthquake time histories, including one with intensity exceeding that of the design level earthquake, are used. Details of the input acceleration motions are given in the following section.

In the inverse triangular load test, the test building is subjected to a full load reversal using a set of lateral forces distributed in an inverse triangular fashion, causing the structure to deform approximately to its first mode shape. The purpose of the inverse triangular load test is twofold. When designing structures either using force-based or displacement-based method, member forces are determined by assuming approximately an inverse triangular acceleration pattern. Hence, response of the building from inverse triangular load tests can be directly compared to the response assumed in the design procedure. The other benefit of the inverse triangular load test is that equivalent viscous damping of the building can be quantified. In the DBD approach, the design base shear is determined using theoretically estimated equivalent viscous damping of the structure as a whole at the design drift level. Since this equivalent damping represents hysteretic energy dissipation occurring during reverse cyclic loads, results from inverse triangular load tests can be also used to experimentally quantify this critical design parameter at different drift levels.

5. EARTHQUAKE INPUT MOTIONS

In the pseudodynamic test, performance of the structure is assessed for a given earthquake input motion. It was felt desirable to subject the test building to several input motions with progressively increasing intensity, thus allowing building performance to be examined at different limit states. However, it was not feasible to compile a suite of suitable acceleration time histories recorded from past earthquakes. Therefore, it was decided that appropriate input motions for pseudodynamic testing be established by modifying recorded earthquake motions on soil type S_c.

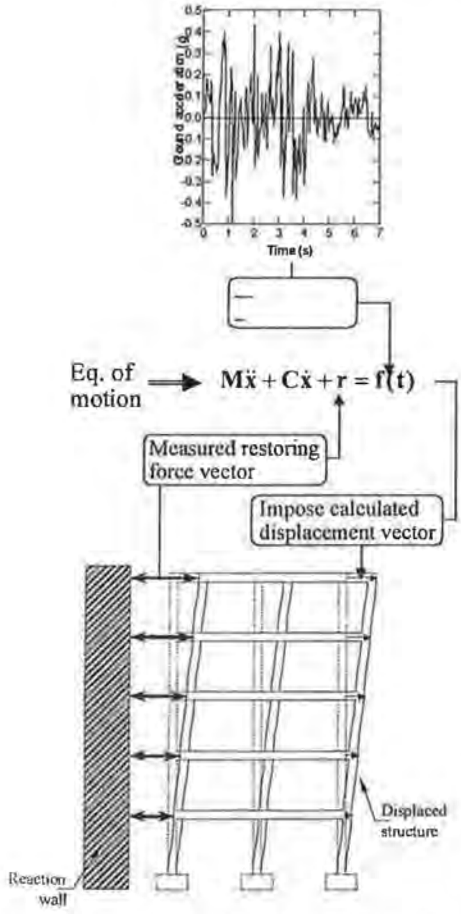

Fig. 10 Pseudodynamic test concept.

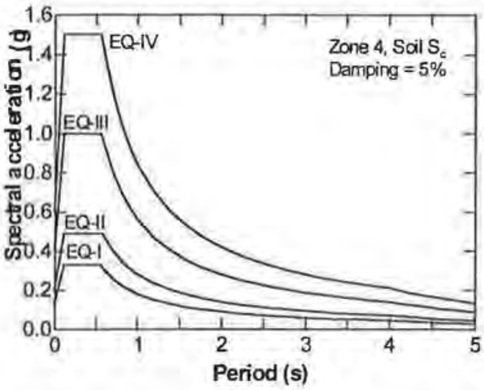

Fig. 11 EQ-I to EQ-IV earthquake hazard spectra.

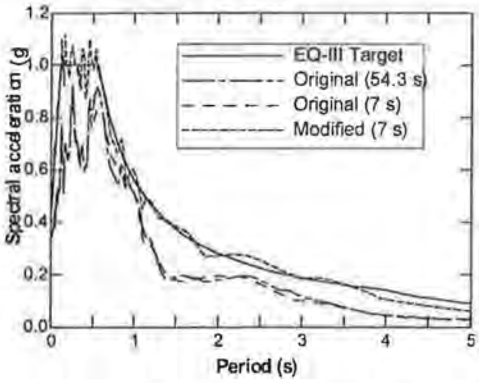

Fig. 12 EQ-III spectrum and 5% damped El Centro acceleration response spectra.

The first step in establishing suitable input motions for pseudodynamic test was to choose a set of target spectra. For this purpose, four levels of performance based spectra as recommended by PBSE Ad-Hoc Committee of SEAOC(8) were used. Using the design spectra in the 1997 NEHRP Provisions(10) as the basis, the PBSE Ad-Hoc Committee recommends acceleration spectra for earthquake hazard ranging from EQ-I to EQ-IV. These four levels represent, respectively, frequent, occasional, rare, and maximum credible earthquakes. In Fig.11, the four level earthquake hazard spectra are shown for soil type S_c, which is one of five soil types classified in the NEHRP provisions. It is noted that the EQ-III level spectrum, which represents design level earthquakes, is identical to the design spectrum included in NEHRP and 1997 UBC codes up to 4.0 s period. At periods beyond 4.0 s, these codes adopt 1/T decay in the spectral values while the spectrum in Fig. 11 reduces spectral accelerations in proportion to $1/T^2$ to maintain constant spectral displacements at longer periods. Accelerations corresponding to the EQ-IV spectrum are intended to be 50% stronger than those of EQ-III (see Fig. 11).

The procedure adopted for obtaining suitable input motions is described here by deriving an EQ-III level input motion from the El Centro record obtained from the 1940 Imperial Valley earthquake (see Table 1). The duration of El Centro record is 53.7 s. As discussed subsequently, a seven-second segment of this record containing the peak acceleration cycle was considered sufficient for pseudodynamic testing. The starting time of all segments, except for EQ-I motion, was decided such that the first peak of each segment is the first peak in the record exceeding 0.1g ground acceleration. For EQ-I level input motion, the same criterion was used with the first peak exceeding 0.05g. Duration of each segment was kept in the range of 4 s for lower intensity motions to 9 s for higher intensity earthquake records with long strong duration

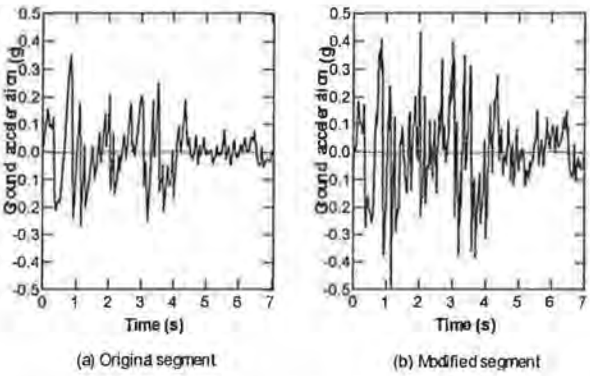

Fig. 13 Seven-second segment of the El Centro record.

In Fig. 12, acceleration response spectra obtained for 53.7 s and 7 s duration of the El Centro record are compared with the EQ-III spectrum. Close agreement of these two El Centro spectra validates the choice of a reduced 7 s duration for the test simulation. The 7 s segment is then modified such that it provides an acceleration spectrum comparable to the EQ-III spectrum. In Fig. 12, it can be seen that the spectrum of the modified motion satisfactorily matches the EQIII spectrum. The necessary modification to the earthquake segment was performed using the program "SHAPE"[11], in which the changes are made iteratively by multiplying Fourier amplitudes of the original motion by spectral ratios established between target acceleration response spectrum and spectrum of the input motion. The original and modified segments of the El Centro records are shown in Fig. 13.

Table 1: Details of the original input records.

EQ Level	Event	Magnitude	Record	Component	PGA
EQ-I	1974 Hollister earthquake	$M_L = 5.2$	Gilroy Array #1	S67W	0.14g
EQ-II	1971San Fernando earthquake	$M_W = 6.6$	Hollywood Storage	N90E	0.21g
EQ-III	1940 Imperial Valley earthquake	$M_W = 6.9$	El Centro	S00E	0.35g
EQ-IVa	1994 Northridge earthquake	$M_W = 6.7$	Sylmar	N00E	0.84g
EQ-IVb	1978 Tabas earthquake	$M_W = 7.4$	Tabas	N16W	0.94g

Four other input motions derived for possible application in the pseudodynamic test of the PRESSS building are shown in Fig. 14, with details of the original records in Table

1. It is noted that in all modified motions, some high frequency content uncharacteristic of natural records is apparent, which elevated the peak ground acceleration (PGA) of the modified records by as much as 50% higher than the target PGA. Low-pass filtering of these records would eliminate the high frequency content and reduce the PGA closer to the target values. However, such filtering was considered unnecessary because the response of the test building should not be sensitive to such high frequency content. Using input motions in the pseudodynamic testing, which closely match the required spectrum but with higher PGAs, will also demonstrate that structural response is not highly influenced by PGA.

6. TEST SEQUENCE

The first step in seismic testing of the PRESSS building is to formulate the stiffness matrix in the uncracked state through a stiffness measurement test. This will be followed by a test sequence consisting of a pseudodynamic test, an inverse triangular load test, and a stiffness measurement test. This sequence is repeated several times with intensity of the input motion for the pseudodynamic test increasing from EQ-I to EQ-IV. At each level of input motion, the inverse triangular load test is performed for one cycle with full reversal such that the resulting maximum positive and negative roof drifts equal the maximum recorded drift in the preceding pseudodynamic test.

Using the above procedure, the PRESSS building will be first tested parallel to the jointed wall system and then in the orthogonal direction to examine the behavior of seismic frames.

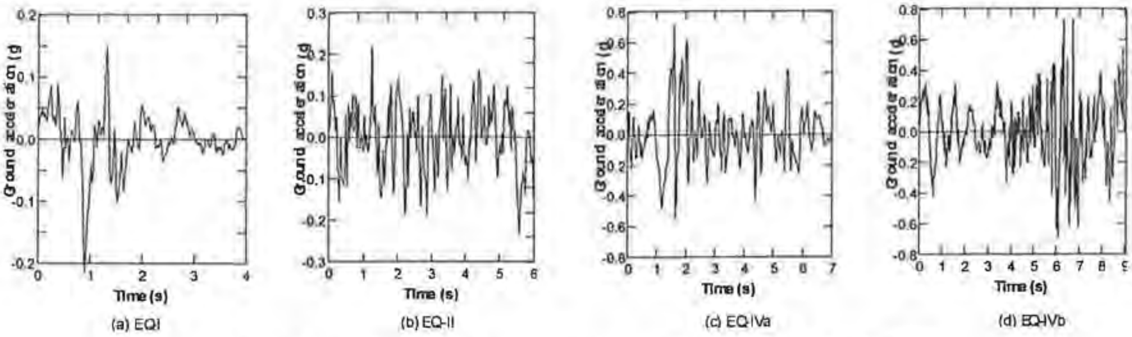

Fig. 14 Four other possible input motions for the PRESSS building test.

7. CONCLUDING REMARKS

A brief description of structural features of the PRESSS five-story precast test building and its seismic test design are presented in this paper. To sufficiently characterize the behavior of the PRESSS building modeling five prototype building systems, three different tests, namely stiffness measurement test, pseudodynamic test and inverse triangular load test, are considered. Significant portions of the test in the two directions are pseudodynamic, in which different level input motions with progressively increasing intensity are used. It is expected that the different levels of pseudodynamic testing together with stiffness measurement and inverse triangular tests will sufficiently quantify the performance of the PRESSS building at different limit states.

Construction and test setup of the PRESSS test building were completed in the Charles Lee Powell Structural Laboratory of the University of California at San Diego by the end of July 1999. Following low amplitude shakedown testing, testing in the wall direction was completed in late August, 1999. Performance of the jointed wall system was excellent, with peak drifts at the design level of seismic intensity being 11% lower than the target drift. There was almost no damage to the structure after being subjected to intensity 50% stronger than the design level input motion, and residual drift, at 7 mm, was only 2% of the peak drift. At the time of writing this paper, the ten loading actuators were being repositioned for testing the building in the frame direction. This testing is scheduled for the first week of September. Preliminary results from seismic testing of the PRESSS building will be presented at the conference.

ACKNOWLEGEMENTS

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RESEARCH MADE POSSIBLE BY UTS SHAKE TABLE FACILITY

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KEYNOTE ADDRESS

The UTS shake table facility was officially opened in October 1996 during a special ceremony attended by the Federal Minister for Science and Technology. The total cost of this acquisition approached one million dollars, including government funding of $583,000. The state-of-the-art table is capable of testing specimens of up to 10 tonnes and can produce any wave form in the frequency range of 0.1 to 50 Hz. It can produce a peak acceleration in the range of 2.5 g (bare table) to 0.9 g (fully loaded) covering a wide range of applications.

This paper highlights some of the special projects undertaken and research made possible by this facility, with the aim of offering it to Australian researchers, engineers, scientists and managers as a "National Facility for Dynamic Testing and Research". It can foster real collaborative and industry focused Research and Development opportunities with mutual benefits for all parties involved.

1. INTRODUCTION :

The 1989 Newcastle earthquake proved once more that Australia is not immune from damaging earthquakes causing significant human and economic losses. Globally, we were again reminded of the devastating effects of severe earthquakes, this time in Turkey with the official death toll at the time of writing this article standing at 12,000 with more than 30,000 people still buried under the rubble.

Such events are reminders to the engineers, scientists and decision makers that seismic risks cannot be underestimated and that any investment in enhancing the community's understanding and preparation for such severe events is a viable one.

In 1993, Four years after the Newcastle earthquake, the researchers at the University of Technology, Sydney and the University of Sydney, were successful in attracting a $300,000 grant under Australian Research Council Mechanism C funding Scheme. The grant provided most of the funds needed to acquire and install a modern shake table facility to facilitate much needed experimental research and foster research and development opportunities with the industry. Identifying a suitable supplier, coupled with the challenging task of constructing a sound foundation for the shaker, took more than 18 months. Figure 1 depicts the massive reinforcement cage prior to pouring the foundation concrete. Figure 2 depicts the completed foundation awaiting the arrival of the shake table.

After a world wide search, MTS Systems corporation of Minnesota, one of the leading producers of advanced dynamic testing equipment, was commissioned to design and fabricate a state-of-the-art shaker incorporating advanced actuator and control technologies. Twelve months later, the complete system was delivered and was subsequently installed and commissioned. Figures 3 and 4 depict part of the installation process.

Although the table was equipped with a modern controller, it lacked much needed data acquisition and analysis software needed to exploit its full potential. To achieve this end, the UTS researchers applied for additional government funding and were again successful in attracting a follow up ARC funding for the amount of $283,000 to equip the facility with the latest analysis software and ancillary equipment. The facility was officially opened in October 1996 during a special ceremony attended by the Federal Minister for Science and Technology. The salient features of the UTS shake table has been reported in an earlier paper and presented at the 1997 Australian Earthquake Engineering Society Conference in Brisbane [1] and hence, a full capability statement will not be given here. It suffices to say that the table is capable of testing specimens of up to 10 tonnes and can produce any wave form in the frequency range of 0.1 to 50 Hz. It can also produce a peak acceleration in the range of 2.5 g (bare table) to 0.9 g (fully loaded) covering a wide range of applications.

This paper will highlight some of the special projects undertaken and research made possible by this facility, with the aim of offering it to Australian researchers, engineers, scientists and managers as a "National Facility for Dynamic Testing and Research". It is hoped that the facility could play its intended role in fostering real collaborative and industry focused Research and Development opportunities with mutual benefits for all parties involved.

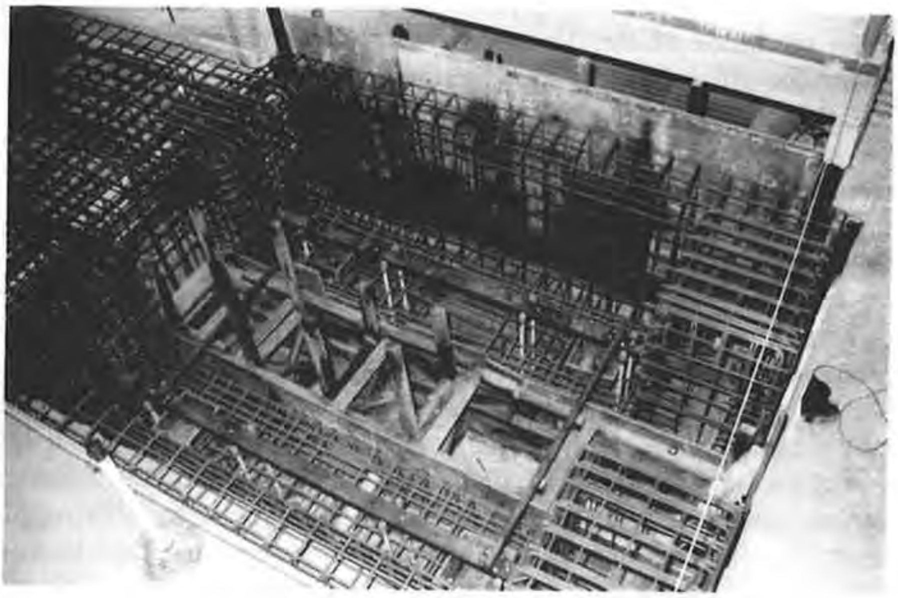

Figure 1: Massive Reinforcement Cage Designed for the Shake Table Foundation

Figure 2: Completed Shake Table Foundation

2. INVESTAGATIVE SEISMIC TESTS FOR CONTROL EQUIPMENT :

The first major project made possible by the newly installed shake table facility was a set of investigative seismic tests performed on control equipment intended for a nuclear power station in north of China. The salient attributes of the tests were reported previously [(2)] with a brief summary given here for the sake of completeness.

The equipment tested is that shown in Figures 5 and 6, respectively. The equipment was subjected to prescribed wave forms along its principal directions as seen in the figures.

An interesting challenge faced during these tests was the need to test the equipment for vertical vibrations using a uni-directional (horizontal) shake table. This challenge was met by fabricating a rigid inclined plane and securing the test specimen to the inclined plane. Testing the equipment in this configuration satisfied the requirements of IEC 980-1989 Standards [3] to subject the test specimen to simultaneous excitations in both the horizontal and vertical directions. This configuration is shown in Figure 7 and is deemed acceptable by the international standards when using a uni-directional shake table. The two accelerometers attached to the base of the specimen, as shown in Figure 8, were to capture the vertical and horizontal accelerations imparted onto the specimen through the inclined plane.

Figure 3: Installation of the Shake Table Bearings

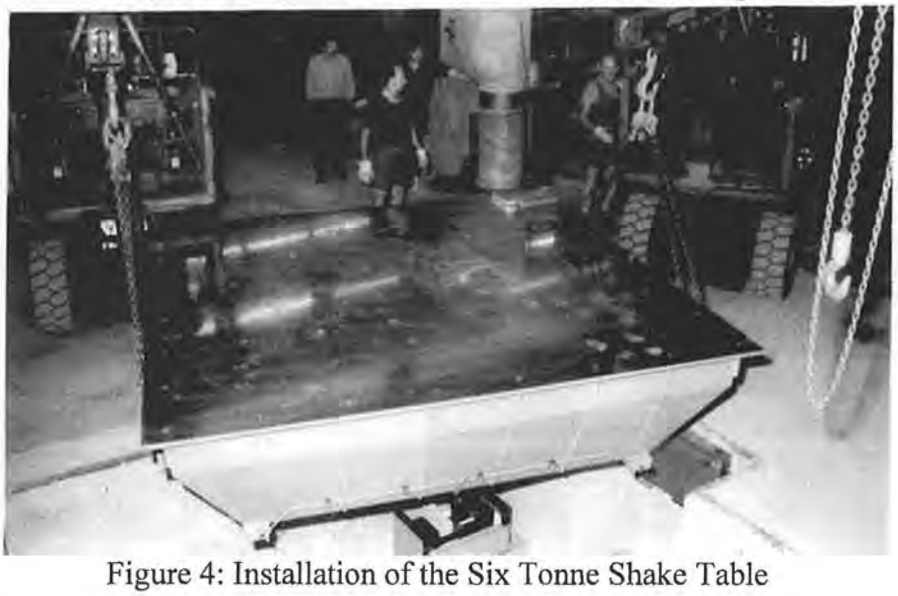

Figure 4: Installation of the Six Tonne Shake Table

3. BEHAVIOUR OF CONCRETE BEAM-COLUMN CONNECTIONS UNDER CYCLIC LOADS :

As part of a collaborative research program with the University of Sydney, UTS and University of Sydney researchers undertook a pilot study into the behaviour of concrete beam-column connections, strengthened by Fibre Reinforced Plastics (FRP) in the joint region, to both static and dynamic loads. The UTS shake table was used to impart cyclic loads to test specimens. In order to compare the behaviour of an unreinforced joint with one reinforced with FRP in the joint region, the tests involved both reinforced and unreinforced specimens. Figure 9 depicts one of the unreinforced specimens connected to the shake table prior to cyclic load testing. The use of the shake table facilitated the test in terms of both its simplicity and speed and proved to be a versatile tool for such tests.

Figure 5: Control Equipment Ready for Seismic Testing (Direction 1)

Figure 6: Control Equipment Ready for Seismic Testing (Direction 2)

Figure 7: Control Equipment Secured to the Inclined Plane

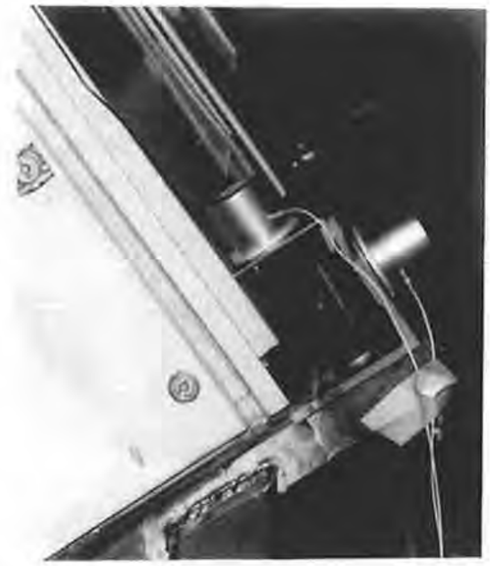

Figure 8: Accelerometers measuring the Vertical and Horizontal Accelerations

4. CYCLIC COMPRESSION AND DYNAMIC SHEAR TESTS OF RUBBER COMPOUNDS :

To address the concerns of an Australian manufacturer to successfully isolate the rotating parts of a mining equipment from its supporting structure, a series of cyclic compression and dynamic shear tests were performed on various rubber compounds produced by the manufacturer. The manufactured rubbers were at the heart of a vibration isolation system for the said equipment. For the rubbers to be effective as isolators, they had to possess very specific material properties at various operating conditions including operations at frequencies approaching 20 Hertz and more.

Once again the UTS shake table facility proved to be the most suitable testing machine to perform the required tests, especially at the higher frequencies where most material testing machines failed to cope with the high frequencies and large amplitude required. One challenge was to replicate service conditions where large rubber pads were compressed evenly to about 6 mm. To achieve this, large pre-compression forces in the range of 35 to 60 tonnes were applied to the rubbers prior to dynamic shear testing. This was achieved by the special design shown in Figures 10 and 11. Figure 10 shows the test apparatus including the shake table as the exciter, while Figure 11 shows a close up of the rubber specimens with the pre compression load acting on them. Dynamic shear tests were performed at frequencies of 1, 10, 15, 20 and 24 Hertz, respectively, producing different load deformation patterns. From the resulting load deflection curves the effect of frequency change on stiffness and damping properties of rubber specimens was evident.

These tests assisted the manufacturer to vary the chemical make up of the rubber compounds in order to achieve the required material properties for an effective vibration isolation system.

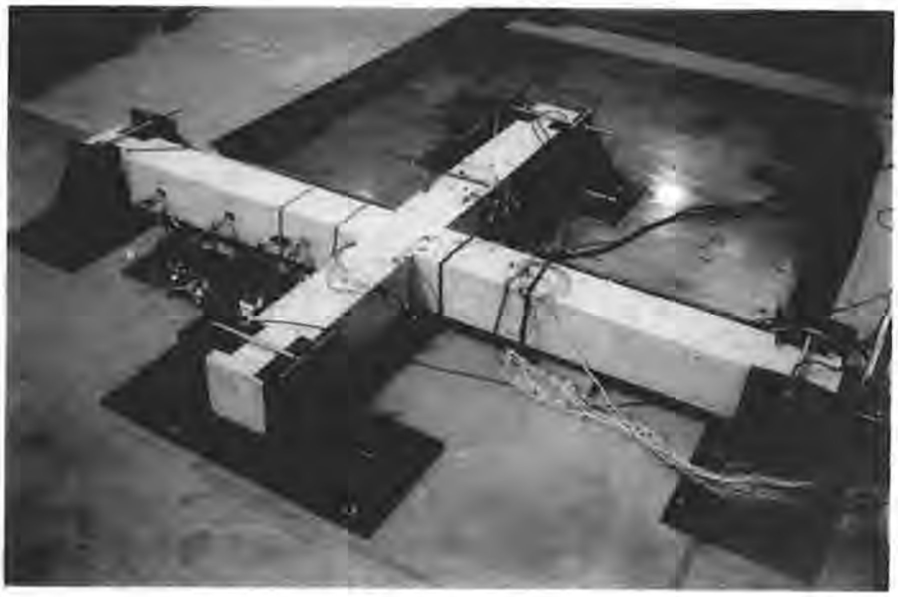

Figure 9: Unreinforced Beam-Column Connections Attached to the Shake Table Prior to Cyclic Load Testing

5. MOTION PERCEPTION TESTS :

The Wind Engineering Group at the University of Sydney, have utilised the UTS shake table facility for a unique research project. The project aims at assessing the human perception of motion associated with wind excited buildings and towers. The other aim of the project is to determine the effect of wind induced vibrations on peoples' performance and productivity well before the motion becomes perceptible and hence objectionable.

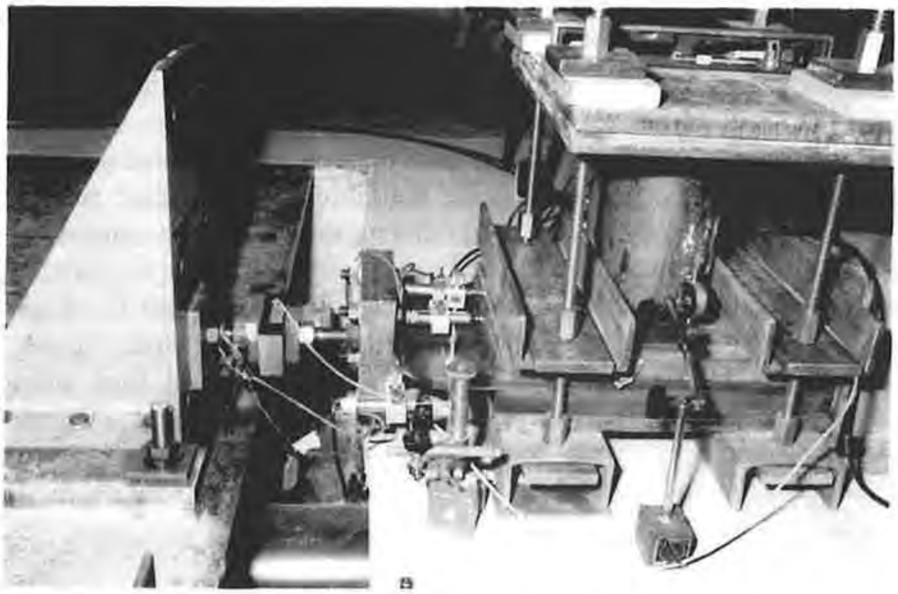

Figure 10: The Test Apparatus for Rubber Test Including the UTS Shake Table

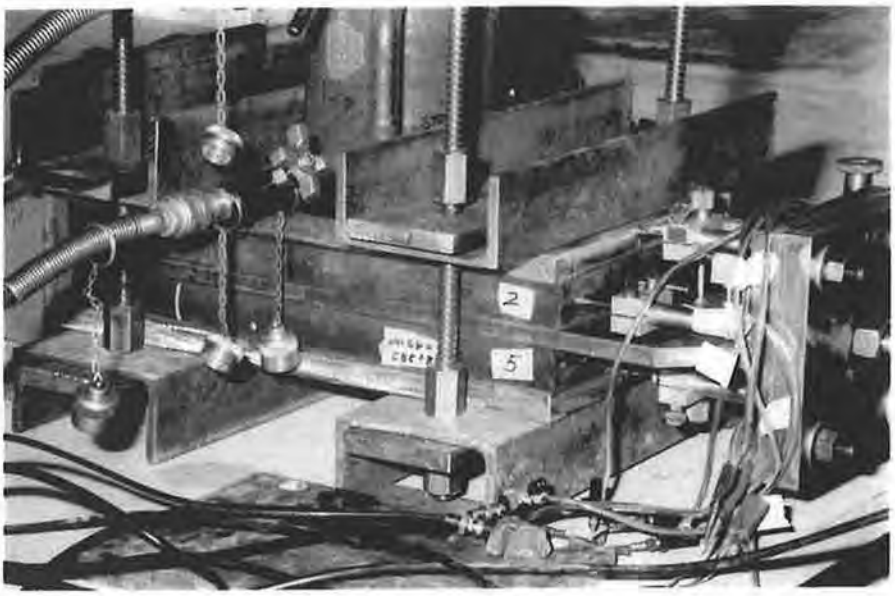

Figure 11: A Close Up of The Rubber Specimens with Pre-Compression Load

The Sydney University team had recorded wind-induced oscillations at the top of a few control towers in Australian airports. The recorded accelerations in digitised form were then used as input to the shake table. An approximately 3m x 3m mobile room, equipped with air conditioning, computer terminals and basic furniture was used to assess the performance and productivity of human subjects under simulated wind conditions. This was achieved by placing the room on the shake table and exposing the subjects to the recorded accelerations. The subjects were asked to perform certain tasks with the shake table off and then on and their performance was later rated. The outcome of this study will guide international standards in setting realistic limits on accelerations as a serviceability criterion for wind excited structures, linked to both motion perception as well as declining productivity stemming from excessive motions. Figure 12 shows one of the participants undertaking a computing task under simulated wind conditions on the shake table.

6. SEISMIC QUALIFICATION TESTS FOR SYSTEM ENCLOSURES:

During an earthquake, telecommunication equipment are subjected to motions that can overstress equipment framework, circuit boards and connectors. In order to ensure the integrity and functionality of such equipment, seismic qualification tests are often required for the equipment installed in seismic regions. The UTS shake table facility was utilised to conduct a series of seismic qualification tests on five different system enclosures, designed for installation in seismic regions. The enclosures are designed and manufactured by Rack Technologies Pty Ltd, a leading Australian company exporting its products to both New Zealand and the United States.

Figure 12: A Human Subject Undertaking a Computing Task under Simulated Wind Conditions on the Shake Table

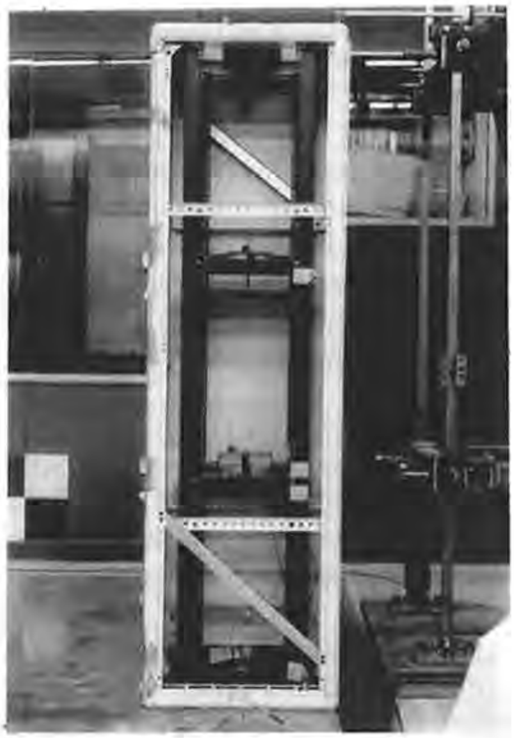

Figure 13: A Typical System Enclosure on the Shake Table Awaiting Seismic Qualification Testing

Test specimens failing to meet the qualification criteria, were structurally modified and strengthened. The modification process was guided by results of modal analysis and modal prediction. The modified enclosures were then subjected to the same qualification tests. The test results show that all enclosures can successfully pass the qualification tests, as is, or with minor structural modification.

Figure 13 shows a typical system enclosure on the shake table awaiting qualification testing. Figure 14 shows a close up of the lower half of the system enclosure with a simulated content weight placed on the lower shelf. A tri-axial accelerometer, recording accelerations at the shelf level can also be seen. The cube shaped accelerometer was placed on top of the simulated content weight as shown. The cold formed diagonal steel member was part of the structural modification to ensure satisfactory performance of test specimens to prescribed excitation. More details on this Research and Development project is given in a separate paper as part of the same Proceedings.

7. INTERNATIONAL BENCHMARKING :

In order to encourage international collaborative research in the area of motion control of building structures, the International Association for Structural Control (IASC) formed a Building Group in 1996 with the aim of devising a few international benchmark building models. The brief was to choose a few benchmark models for both analytical and experimental research in the field of Structural Control. Adoption of a few benchmark models will allow researchers, all over the world, to test their control algorithms on the same models possessing identical properties. This will allow direct comparison of results among researchers with obvious benefits.

In particular, the IASC, was keen to develop a versatile experimental benchmark model for shake table testing. The model had to be of a size and scale to allow implementation of reasonably sized control devices such as Tuned Mass Dampers, Active Mass Drivers, Liquid Dampers, Active Tendons, Visco-elastic Dampers, Semi-Active Dampers and others onto the model in order to verify the feasibility of employing control devices to suppress wind and earthquake induced motions in building structures.

The UTS researchers, in collaboration with researchers from the University of California, Irvine, were entrusted with the task of designing and fabricating a versatile building model as one of IASC experimental benchmark buildings. In 1998, the design and fabrication of the model was completed and the model is now offered to the international research community for testing and analysis.

The experimental benchmark model (shown in Figure 15) offers the flexibility needed to model and test various building configurations. Its innovative connection details allow variable floor heights. Replaceable column-beam connections allow modelling the building with both fixed and pinned connections. The mass of the model can vary between 750 kg to nearly 2 tonnes by providing additional mass to it in the form of plates and/or point masses at pre-designated locations. The model is currently undergoing several tests to determine its dynamic characteristics for various combinations of height, mass, joint fixity and direction of loading following a thorough System Identification exercise. The model is three meters high with a footprint of 1.5m x 1.0m. It consists of two bays in one direction and a single bay in the other. The benchmark frame is already regarded as an invaluable research tool for the ever active area of Structural Control.

Figure 14: A Close up of the Lower Half of the System Enclosure

Figure 15: UTS Experimental Benchmark Frame

8. CONCLUDING REMARKS:

Since its opening in October 1996, the UTS shake table facility has been instrumental in ensuring the successful completion of several applied and contract research projects. The UTS dynamic facility, not only enjoys a modern and well built shake table with a modern actuator and control system, is also equipped with state-of-the-art hardware and software systems. Among which are: state-of-the art 32 channel high speed HP Vxi analyser with on-board DSP to improve total system performance and LMS CAD-X dynamic test lab software, regarded as one of the best and most powerful tools for data acquisition, modal analysis and modal prediction in both “time” and “frequency” domains. The software is capable of performing modification, prediction and sub-structuring tasks associated with complex problems requiring testing of various design and analysis alternatives.

The UTS research team offers this unique and state-of-the-art facility to all researchers, scientists, engineers and decision makers in Australia and overseas and looks forward to to collaborative research and consulting opportunities with mutual benefits to all parties involved. It stands ready to serve the community, in every possible way, in meeting the challenge of eradicating the term "seismic calamity" from our vocabulary.

9. ACKNOWLEDGEMENT :

The author wishes to acknowledge the continuous dedication and fine services rendered by Dr J Li in managing the day to day operations of the shake table facility, the efforts and commitment displayed by Mrs Yimin Wu in performing pre and post test analyses, and the valuable services rendered by Mr J Holmes in producing the high quality photographs included in this paper.

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MASONRY CONSTRUCTION - HAVE WE LEARNT ANYTHING FROM THE NEWCASTLE EARTHQUAKE ?

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The large amount of damage sustained by masonry in the Newcastle earthquake has prompted significant research and other activity in the past ten years in an attempt to address many of the issues raised. A new masonry standard has been produced with specific provisions for seismic design, and research into various aspects of the seismic behaviour of URM has taken place (including the behaviour of slip joints and joints containing dpc membranes, wall tie performance and, bond strength studies). The bulk of this research has been carried out by the Masonry Research Group at the University of Newcastle. This paper gives an overview of the above aspects and past and current research, and also questions whether the suggested improvements in masonry construction are actually being implemented in practice.

1. INTRODUCTION

A large proportion of the damage in the 1989 Newcastle Earthquake was sustained by unreinforced masonry. When examined in detail, it was clear that a large proportion of the damage was avoidable and the result of factors such as the lack of consideration of earthquake loads in design, soft storey effects, lack of tying and support, poor workmanship, durability problems, lack of adherence to existing codes, and simply bad design and detailing. Well designed and constructed masonry structures performed satisfactorily in many instances. The large amount of damage to masonry has prompted significant research and other activity in the past ten years in an attempt to address many of the issues raised. The bulk of this research has been carried out by the Masonry Research Group at the University of Newcastle. This paper gives an overview of the above aspects and on-going research, and also questions whether the suggested improvements in masonry construction are actually being implemented in practice.

2. PERFORMANCE OF MASONRY STRUCTURES IN THE NEWCASTLE EARTHQUAKE

The performance of structures in the Newcastle earthquake has been well documented (1-3) and will not be discussed in detail here. One of the main features was the widespread damage sustained by unreinforced masonry. However, when the damage is examined systematically, it is apparent that a significant proportion of the damage could have been avoided if even rudimentary seismic design requirements had been in place and the masonry had been designed, detailed and constructed in accordance with the appropriate codes (4). Areas directly related to earthquake design requirements (and now covered by AS1170.4) included: lack of clearly defined load paths in buildings; soft soil effects; building layout in plan and elevation; inadequate support of free standing elements such as parapets and gable ends; lack of support of internal non-loadbearing walls; and damage to masonry cladding and/or infill in framed construction.

Areas directly related to masonry design, detailing and construction have been described in detail elsewhere (5), and included: masonry quality, particularly low bond strength; masonry durability, in particular corrosion of wall ties and fitments, and general deterioration of older structures; failure of masonry under face loading due to lack off, or inadequate tying; and slip on membrane type damp-proof courses.

It is clear from the above that a large proportion of the damage could have been avoided by correct design and detailing practices. The satisfactory performance of a number of well engineered loadbearing masonry structures attests to this fact. There are obviously a number of lessons to be learnt from the Newcastle earthquake, and from a masonry perspective it is interesting to examine subsequent developments that have taken place.

3. SUBSEQUENT DEVELOPMENTS IN THE MASONRY FIELD

In the ensuing 10 years since the Newcastle earthquake, there has been significant activity in the masonry area to address many of the issues described above. These have taken place in parallel with the development and publication of the revised earthquake loading standard AS1170.4. Some of this activity is described below.

3.1 Masonry Standards

A new edition of the Australian Standard for Masonry Structures (AS3700-1998) was published recently. This came 10 years after the previous edition and reflected both the advances in the state of knowledge of the behaviour of masonry structures, as well as the increased emphasis on seismic performance. Some of the changes and additions have resulted directly from research following the Newcastle earthquake.

In parallel with the review, there has been an attempt by Standards Australia to bring all the associated codes into phase with the main document. To a large extent this has been successful with the recent or imminent publication of codes for Masonry Units and Test Methods, Wall Ties and Accessories, and Strengthening of Existing Buildings for Earthquakes. A "deemed-to-comply" Standard for masonry housing is also well advanced. Several of these codes are combined Australian-New Zealand Standards in line with the harmonisation policies of the two countries. Some of the AS3700 changes include:

- an appendix on earthquake design. The purpose of this Appendix is to enable the seismic performance of the wide variety of masonry construction permitted by AS3700 (unreinforced masonry, masonry of mixed construction, wide-spaced and close-spaced reinforced masonry) to be determined in the context of the requirements of AS1170.4).
- increased durability requirements, particularly for ties and fitments in coastal environments.
- guidance on the shear design of joints containing damp-proof course membranes or slip joints.
- a rational method for wall tie design.
- a section on prestressed masonry.
- clearer requirements on workmanship, tying and detailing etc.

3.2 Shear Capacity of Slip Joints and Joints Containing Membrane Type Damp-Proof Courses (dpc's)

With the exception of roof connections, earthquake induced forces in loadbearing masonry structures can be transmitted through wall-floor connections and other potential slip planes by friction. For adequate performance, reasonable levels of precompression are obviously required to prevent slip occurring.

Over the last few years there has been extensive research aimed at establishing the shear characteristics of joints of this type. The bulk of this work has been carried out at the University of Newcastle in collaboration with the University of Adelaide and the Association of Consulting Structural Engineers of NSW. A series of static, cyclic and dynamic tests have been carried out on a range of joint types to assess their shear performance under both short term and long term loading. This work is still in progress, but interim recommendations for design have been made (6) and design values incorporated in the new masonry standard. This will enable designers to more easily establish and confirm load paths in loadbearing masonry structures. Current research in this area is concentrating on the performance of these joints under sustained long term loading, and in particular their ability to creep and thus compensate for concrete shrinkage and brick growth.

3.3 Wall Ties

Wall ties are an essential component of a masonry structure, as in most cases they are the prime means of support for masonry veneers. They must possess adequate durability to maintain their integrity for the life of the structure, and be able to transmit the appropriate tensile and compressive forces induced by face loading from wind or earthquake. Recent research has resulted in the development of rational methods for wall tie design (7), as well as a test for assessing the corrosion resistance ratings of corrosion protection systems (this test is incorporated in the new wall tie standard). A study is also in progress to assess the seismic performance of typical Australian wall tie details for both cavity and veneer constructions (8). Cyclic tests on typical assemblages indicate that conventional cavity ties and side fixed veneer ties perform reasonably well. Face fixed veneer ties and several of the "clip on" type veneer ties for steel stud construction fail prematurely after only a few tension - compression cycles. In this case, more positive screwed connections may be required.

3.4 Masonry Quality

One of the major problems in masonry construction is the achievement of adequate bond strength and mortar durability due to widespread lack of adherence to the mortar and workmanship provisions of AS3700. Common abuses include inaccurate mortar batching (leading to lack of cement and subsequent durability problems), the omission of lime from the mix, and overdosing with plasticisers or workability agents leading to drastic reductions in bond strength. An additional recent worrying trend is the increasing use of "bricklayers clay" in lieu of lime, to enhance the workability of a mix made with poorly graded sand. These practices have particularly detrimental effects on the durability of the mortar and the masonry bond strength, both pivotal to the effective long term performance of the structure. Extensive research is being carried out at the University of Newcastle in conjunction with industry and CSIRO on the both the applied and fundamental aspects of bond strength to establish the basic mechanisms of bond creation and identify the most crucial parameters. This work has already identified the detrimental effect of air entraining agents on bond strength (9).

4. THE REALITY

Construction: Despite the improvements and advances in the state of knowledge that have been described above, it is a matter of extreme concern that the standards of masonry design, construction and detailing have not necessarily improved, particularly in the housing sector. There is still widespread evidence of mortar abuse, poor workmanship, and the installation of tying systems which do not conform to the requirements of AS3700. The seismic performance of these structures must therefore be suspect.

Supervision: There is a general lack of supervision and inspection of masonry construction. This is often because masonry is not considered as part of the design brief (or the fee structure) of the structural engineer. Consequently, masonry aspects are left to the architect and/or building surveyor and the bricklayer who do not necessarily have an appreciation of the importance of producing good quality masonry. In the earthquake context, there is no such thing as "non-structural' masonry, and the importance of adequate support and tying of *all* masonry is not necessarily appreciated by the project supervisor and/or the statutory authorities.

Education: A fundamental problem is the relative ignorance of masonry behaviour on the part of builders, architects and even structural engineers in some cases. This is primarily the result of the lack of inclusion of masonry material in undergraduate architecture and engineering curricula. Knowledge therefore has to be progressively acquired in industry after basic training, and this can result in the acceptance and continuation of suspect practices. The clay brick industry, through the Clay Brick and Paver Institute, is attempting to address this challenge by developing a series of teaching packages for use in engineering and architecture schools. The first of these has just been released and distributed to all Civil Engineering Schools in Australia as a teaching resource for potential incorporation in the final stages of the curriculum.

5. SUMMARY AND CONCLUSIONS

In the 10 years since the Newcastle earthquake there have been major developments and changes which will allow the production of unreinforced masonry structures capable of withstanding small to moderate earthquakes without excessive damage. The principal concern remains the quality of the masonry as constructed, since there is widespread anecdotal evidence that suspect detailing and construction practices are still endemic. This problem can only be remedied by appropriate education, site supervision and greater involvement of the structural engineer in the design and construction of masonry.

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MASONRY RESEARCH IN AUSTRALIA SINCE NEWCASTLE - 10 YEARS AND WHAT HAVE WE LEARNED ?

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John Wilson is Chairman of the Board of Engineering, Victorian Division of the Institution of Engineers, Committee member of the Australian Earthquake Engineering Society, and Member of the Australian Standards Committee for Earthquake Loading. He was Senior Engineer with Ove Arup and Partners before becoming Senior Lecturer at The University of Melbourne in 1992. He is co-author of a book and numerous publications in many different areas of earthquake engineering and structural dynamics.

ABSTRACT:

The 1989 Newcastle earthquake raised awareness in Australia of the risk posed to unreinforced masonry (URM) buildings by earthquakes and prompted a flurry of research activity in this area. This paper presents an overview of the research conducted in Australia since 1989 on the topic of the seismic behaviour of URM buildings. The paper highlights the questions that have answered, those still outstanding, and the practical outcomes that have made their way into practice through codes and their like as a result of this collective research effort.

Paper No. 4

1. INTRODUCTION

In December 1989 Newcastle experienced a magnitude M = 5.6 earthquake. That unexpected seismic event caused over $1 billion damage and, for the first time in Australia, 13 deaths due to earthquake. Intensive post-disaster investigations consistently highlighted the fact that [10]:

- most of the damage was associated with URM construction on soft soil sites;
- many of the damaged URM buildings were badly deteriorated, poorly constructed or designed; and
- many of the failure modes observed in Newcastle could have been prevented with simple improvements to detailing practice.

From this starting point, there has been a significant amount of research into the seismic resistance of URM construction. In the remainder of this paper, we will attempt to briefly summarise the results of this research in Australia over the past 10 years with particular attention being paid to the work carried out by the co-authors at the Universities of Adelaide and Melbourne.

2. OVERVIEW OF MASONRY RESEARCH IN AUSTRALIA

General Seismic Behaviour

Following the 1989 earthquake, there was a real need to be able to explain why URM buildings fared so well and others fared so poorly. In order to understand the damage pattern, fourteen URM buildings in Adelaide were analysed as part of a large case study into the seismic resistance of typcial URM buildings [5]. This project aimed to to (1) establish the accuracy of the period formulae in AS1170.4 for URM buildings; (2) determine the dynamic in-plane stiffness of unreinforced brick wall panels; and (3) evaluate the current earthquake design requirements in AS1170.4 for URM buildings. The project consisted of three phases. In the first phase, ambient vibration field tests were conducted on 14 URM buildings in Adelaide to verify the suitability of the period formulae in AS1170.4. In the second phase, earthquake simulator testing of URM wall panels was conducted to study the in-plane dynamic behaviour of clay brick masonry walls. From these tests, the effective dynamic stiffness of clay brick walls was calculated. In the final phase, the detailing and earthquake force requirements of AS1170.4 were evaluated against the results of detailed 3D dynamic analyses of eleven of the Adelaide buildings studied in phase one of this project. The study resulted in 2 major findings.

(1) Current design methods overestimate the stiffness of brick walls by a factor of as much as 10. Furthermore, the value for Young's modulus needed to correctly predict the fundamental period of the buildings was found to be about 1000 MPa, much less than normally expected for good quality URM (ie, about 5000 MPa). Similar results were found from tests conducted at Melbourne University [7] and in Sydney at UTS [3]. Of course, these estimates of seismic demands led to the need to verify the "seismic capacity" of URM walls and connections.

(2) The minimum earthquake connection force requirements in AS1170.4 are significantly smaller than required, implying that URM buildings must rely on friction to withstand earthquake loading. Roughly half the buildings did not comply with the current seismic design requirements and so are theoretically at risk. The

seismic demands for connections in URM buildings were calculated for each building in this study. It was noted that in the absence of mechanical fasteners to "tie" the walls of buildings to the floor and roof systems, friction would be required to transmit the seismic forces generated by the earthquake. For most buildings in this study, a "capacity" friction coefficient of 0.3 was all that was required.

Masonry connections

Many walls failed during the Newcastle earthquake due to the wall ties which were either insufficient in number, missing completely, or had been severely degraded over time. Furthermore, the new Australian earthquake loading code AS 1170.4 required that all parts of buildings be positively "tied" together. Hence, research was required to better understand the seismic forces placed on wall ties and to quantify the seismic "capacity" of typical friction joints in URM buildings. Experimental and analytical work at Newcastle has greatly improved our understanding of wall tie behaviour under earthquake loading in both brick veneer and brick cavity construction [4]. With regard to wall-to-floor connections, initial static tests at Newcastle University [12] were performed to establish reliable friction coefficients for damp proof course (dpc) and slip joints in typical brick masonry construction. These results were subsequently confirmed with quasi-static cyclic tests at Newcastle and dynamic shake table tests at Adelaide University [2] and are consistent with results of quasi-static tests conducted at the University of South Adelaide [13]. These results indicated that typical dpc joints, through friction, are capable of transferring shear force greater than 30% of the corresponding normal force at the dpc joint.

Out-of-plane wall behaviour

Another common failure mode in Newcastle was out-of-plane bending failure of walls. Research on this topic has been carried out by a number of investigators. For example, an improved method for calculating the flexural strength of brick masonry has been incorporated in the new version of AS 3700. This method is based on virtual work principles and relies on realistic estimates of the moment capacity of brick masonry along vertical and diagonal cracks. Expressions for these values are largely empirical and are given in AS 3700. There is still a need to improve our understanding of the influence of vertical compressive stress on the moment capacity of URM walls along vertical and diagonal crack lines.

Much work has also been conducted to establish, through dynamic testing at Adelaide and Melbourne Universities, the flexural strength of brick walls. The shaking table tests results indicate that the walls have a surprising amount of residual capacity beyond the "first crack" loading. An analytical model was originally developed by Melbourne researchers [8] to describe the dynamic behaviour of URM parapet walls. This model has been further developed to accurately predict the time-history response of URM walls simply supported at top and bottom [1] subject to arbitrary dynamic loading, including earthquake ground motion.

The analytical results further indicate that it is displacements which are critical for establishing seismic capacity of URM walls. Indeed, these results have been used to evaluate the acceleration-based, the velocity-based and the displacement-based analysis procedures. Acceleration-based force design criteria appear to overestimate the likely seismic demand placed on a URM wall. The findings suggest that a displacement-based approach for design of brick walls in bending may be more accurate and less conservative than current force-based design procedures. However, only 1-way (vertical) bending tests have been conducted so far. A substantial amount of effort is required to extend the results to the much more representative case of biaxial bending.

In-plane wall behaviour

Researchers have also worked to fine-tune their models of the ink-plane behaviour of brick masonry walls. Zhuge [14] has used the results of experimental work at Sydney [3] and Adelaide [6] to validate finite element models for this problem. Klopp at Adelaide used standard finite elements with a Young's modulus of 1065 MPa to analyse the buildings in his case study. Experimental and analytical work is currently underway at Newcastle to study the in-plane behaviour of masonry panels subjected to dynamic cyclic biaxial loading [11]. The results of these tests will provide researchers with much needed data for validation of improved analytical models.

3. WHAT DO WE KNOW NOW THAT WE DIDN'T IN 1989

Many of the questions arising out of the Newcastle earthquake were associated with why some buildings survived and why other (apparently similar) buildings failed. So, what have we learned?

- We now have a much better idea of how to model URM buildings for earthquake analysis and design. There remains some discrepancy in the literature as to what are realistic values for Young's modulus and masonry stiffness. Continuing work in this area is expected to further clarify these questions.
- What we did not fully appreciate before the Newcastle earthquake was how widespread the wall tie related problems were. To industry's credit, they quickly developed a variety of "rust-resistant" alternatives. Furthermore, we now have a much better appreciation of the force distribution between wall ties in both veneer and cavity forms of construction. More rational specifications of wall tie spacings are now possible because of this.
- Based on many case studies, we now have a quantitative understanding of the seismic demand for most of the key structural components and connections in the seismic load-path for URM buildings. This information will enable engineers to better design for earthquake effects.
- We now have an improved (static) method for out-of-plane strength of URM based on virtual work principles. While the method still relies on largely empirical data for the moment capacity of brickwork along vertical and diagonal crack lines, it is much more accurate than previous linear elastic based methods.
- Furthermore, we now have a better understanding of the physical process governing the out-of-plane collapse resistance of URM walls. The displacement of the wall support in space appears to control the response behaviour of the wall, which provides an explanation to the severe damages of URM buildings on soft soil sites.
- Finally, and perhaps most importantly, there is a raised awareness amongst engineers, architects and builders of the need for seismic resistance in all buildings.

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MODELLING OF THE EARTHQUAKE GROUND MOTIONS GENERATED BY THE NEWCASTLE EARTHQUAKE

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ABSTRACT:

No strong motion accelerograms have been recorded from the mainshock of the Newcastle earthquake. Methodologies developed recently at the University of Melbourne have been employed to model earthquake ground motions generated by the earthquake to compare with macro-seismic data. The developed procedure consists of two components: (i) Prediction of bedrock motions based on a seismological source model and a regional crustal model (ii) Prediction of the effect of soil resonance on the response spectra. The acceleration, velocity and displacement parameters obtained from the procedure for a number of damage sites in Newcastle have been tabulated.

Paper No. 5

1. INTRODUCTION

A magnitude 5.5 earthquake which struck Newcastle, New South Wales, Australia in December 1989 caused widespread damages to infrastructures and resulted in 11 deaths. No strong motion accelerograms have been recorded from the mainshock of the earthquake and there were speculations over the attenuation and amplification of the earthquake ground motions. Significantly, the predominant geological feature that influenced the extent of seismically induced damages to infrastructures was the variation in the depth of the alluvial material overlying bedrock.

Methodologies developed recently at the University of Melbourne have been employed to model earthquake ground motions generated by the Newcastle earthquake. The developed procedure consists of two components: (i) Prediction of bedrock motions based on a seismological source model and a regional crustal model. (ii) Prediction of the effect of soil resonance on the response spectra. The acceleration, velocity and displacement parameters obtained from the above procedure for a number of damage sites in Newcastle have been tabulated.

2. SEISMOLOGICAL MODELLING OF BEDROCK MOTIONS

Recent seismological investigations indicate that the regional dependent frequency characteristics of earthquake ground motions are largely attributed to regional variations in the crustal properties. In comparison, the effect of regional variations is much more moderate in the average source properties of the generated seismic shear waves(1). The two generic crustal conditions, named as generic "hard rock" & "rock", have been defined in the context of a seismological model references 1 & 2.

The maximum response spectral displacement (RSD_{max}) predicted for the generic "hard rock" crustal conditions can be approximated by the following expressions, which have been derived from stochastic simulations of the seismological model(3):

$$RSD_{max} = 14\ \alpha_D(M)\ \beta(R)\ \gamma_D(M,R) \qquad (RSD_{max} \text{ in mm}) \qquad (1)$$

$$\text{where } \alpha_D(M) = 0.20 + 0.80\ (M-5)^{2.3} \qquad (2a)$$

$$\beta(R) = 30 / R \qquad (R \text{ in km}) \qquad (2b)$$

The crustal factor $\gamma_D(M,R)$ is an amplification ratio which accounts for the different crustal conditions(3) (It is recommended that in the absence of reliable information on the crustal parameters that "rock" crustal conditions be conservatively assumed.):

$$\gamma_D(M,R) = 1.6 + (30-R)/200 + (6-M)/10 \qquad \text{(for generic "rock")} \qquad (3a)$$

$$\gamma_D(M,R) = 1 \qquad \text{(for generic "hard rock")} \qquad (3b)$$

Parameters defining velocity and acceleration properties can also be predicted by similar expressions(3). The response spectrum modeling approach is the direct determination of the acceleration, velocity and dsiplacement response spectra (RSA, RSV and RSD respectively) based on the ground motion parameters described above(3).

3. THE BEDROCK MODELLING OF THE NEWCASTLE EARTHQUAKE

There were no strong motion accelerograms in the Newcastle area at the time of the earthquake. Thus, the seismological model has been employed to model the Newcastle Earthquake. The model has been checked by comparing with peak acceleration and velocity recorded from the nearby Ellalong Earthquake[(4)]. [Figures 1 & 2]

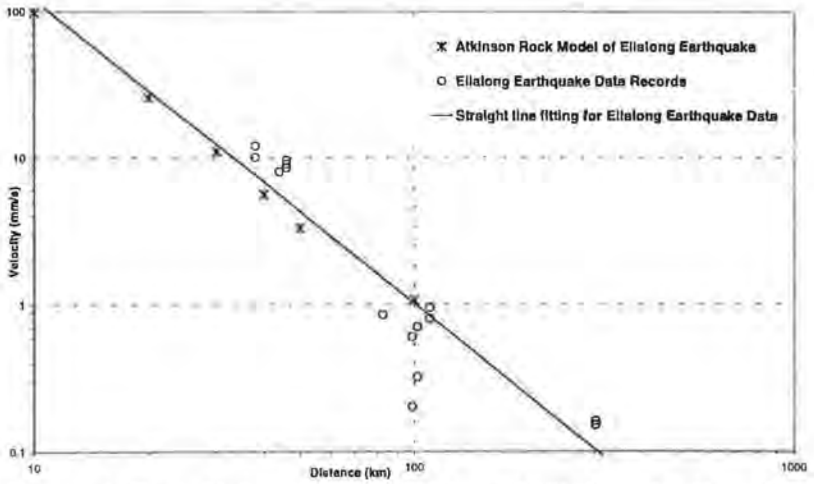

Figure 1. Peak Velocity of Rock Sites of Ellalong Earthquake.

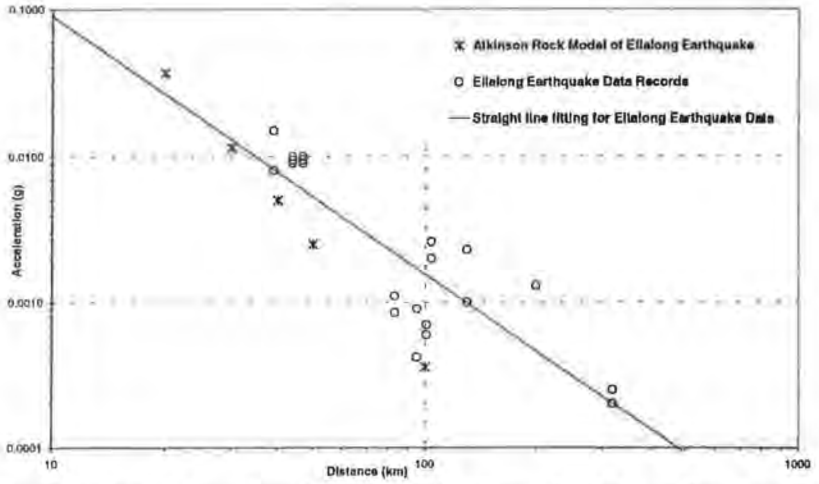

Figure 2. Peak Acceleration of Rock Sites of Ellalong Earthquake.

The Ellalong Earthquake of magnitude 5.3, occurred 3-4km west of Ellalong at the town of Cessnock, 50km west of Newcastle on 6th August 1994[(5)]. Since the earthquake was very shallow (depth ~ 1 to 2 km), it has been assumed that the entire wave travel path was within the upper crust region which possessed very low attenuation properties as inferred by the sesimological model[(2&6)].

By utilising the existing Atkinson rock model with modification to the parameters to simulate the Ellalong Earthquake, the recorded peak velocity and acceleration have been compared with the simulated data. [Figures 1&2] The good agreement of the seismological model with field measurements confirms the accuracy of equations (1-3) in predicting ground motion parameters for rock sites and bedrock.

4. FRAME ANALOGY SOIL AMPLIFICATION MODEL FOR SOIL SITES

The procedure developed recently by the authors to model the effect of soil resonance is known as the "Frame Analogy Soil Amplification" (FASA), as it is based on drawing an analogy between the dynamic response of a soil column and a moment resisting frame developing "beam-sway" mechanism[3&7], to predict the peak ground acceleration (PGA) and the maximum response spectral acceleration (RSA_{max}) on soil sites. According to the FASA model, RSA_{max} is proportional to the RSA of the bedrock response spectrum (Sa_i) at the natural period of the site (T_i, which is also the corner period of the soil response spectrum). Further details of the FASA procedure for modelling acceleration response spectra is described in reference 7. The level of PGA and RSA_{max} on a soil site depends largely on both T_i and the shape of the characteristic response spectrum of the underlying bedrock. The key equations in the FASA model are summarized in Figure 3.

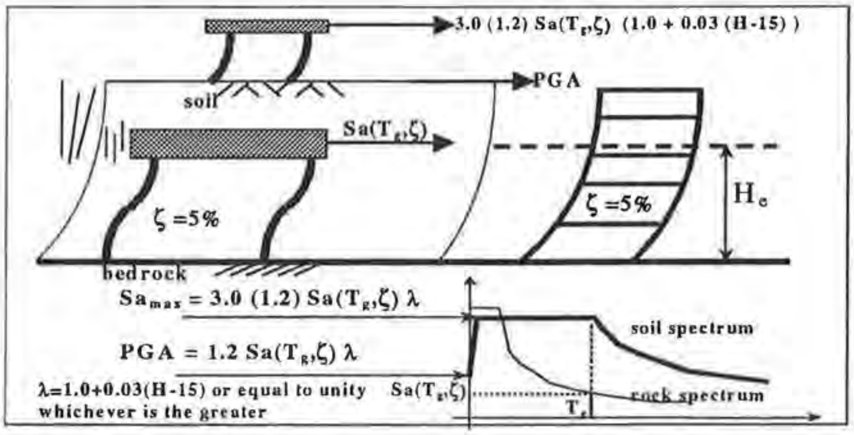

Figure 3.
The Frame Analogy Soil Amplification (FASA) Model.

Table 1. Site prediction using FASA model for different sites in Newcastle Earthquake

Site	Ti (sec)	Tg (sec)	RSA_{max} (g)	RSV_{max} (PGV) (mm/sec)	RSD_{max} (mm)
Newcastle Workers Club	0.55	0.82	0.337	431 (215)*	56.3
Hamilton – James & Murray Sts.	0.51	0.78	0.386	467 (234)*	57.6
Hamilton - Francis Xavier	0.50	0.76	0.541	642 (321)*	77.7
Newcastle RSL	0.22	0.43	0.940	636 (318)*	43.9
Franklins (old store bldg)	0.25	0.47	0.860	630 (315)*	47.0
Newcastle Technical College	0.55	0.68	0.486	517 (259)*	56.2
Taxation Office	0.16	0.34	1.057	560 (280)*	30.2
Tudor Inn Motel	0.18	0.37	1.011	582 (291)*	34.2
Ambassador Hotel	0.77	0.89	0.420	586 (293)*	83.2

* Values in brackets are the implied peak ground velocity (PGV), for PGV ~ RSV/2.

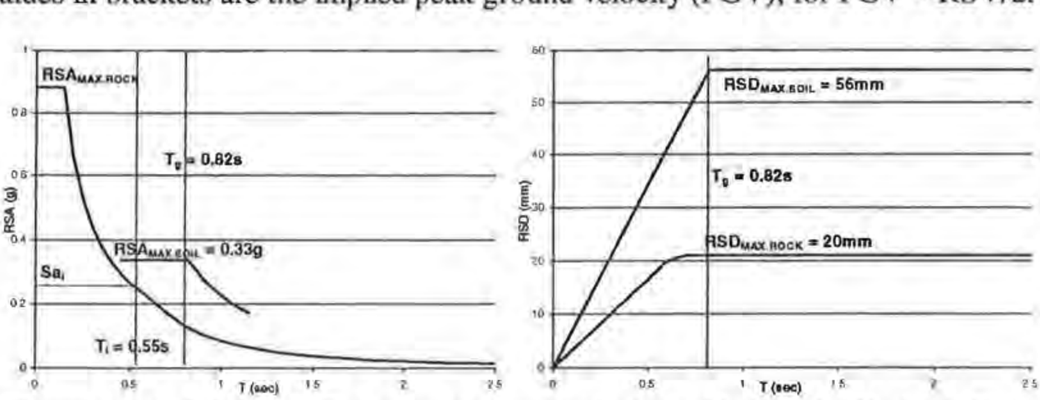

Figure 4 & 5. RSA & RSD for bedrock and soil site at Newcastle Workers Club.

Figure 4 & 5. RSA & RSD for bedrock and soil site at Newcastle Workers Club. Using the FASA model, the RSD_{max} and RSA_{max} of soil sites in Newcastle have been estimated to range between 30 and 83mm and 0.3g and 1g respectively. [Table 1] The rock RSD_{max} and Sa_{max} are about 20mm and 0.8g respectively. [Figures 4 & 5]

According to the Australian Standard AS 1170.4[8] the acceleration coefficient (a) has been specified as 0.11g for the Newcastle region (which corresponds to a PGV of 82mm/s). Clearly, this is much lower than the 220~320mm/s predicted by the seismological model and the FASA model. [Table 1] Significantly, the very high predicted PGV level is consistent with speculations by McCue *et al* [1995] based on extrapolations from the Modified Mercalli Intensity data.

5. CONCLUSIONS

1. Predictions of the ground motion parameters by the seismological model are in good agreement with field measurements from the Ellalong Earthquake.
2. The maximum response spectral acceleration, velocity and displacement have been estimated for the 9 Newcastle soil sites by the seismological model and the FASA model [Table 1].
3. The predicted PGV in this study significantly exceed current code specifications. However, they were in good agreement with speculation based on extrapolation from the Modified Mercalli Intensity data.

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MODELLING THE COLLAPSE BEHAVIOUR OF UNREINFORCED MASONRY WALLS DURING THE NEWCASTLE EARTHQUAKE

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ABSTRACT:

The Magnitude 5.5 earthquake which struck Newcastle, New South Wales (NSW) Australia in December 1989 was characterised by widespread damage to unreinforced masonry (URM) walls mostly failing in out-of-plane bending. Damage surveillance identified, amongst other factors, that soft soil sediment sites could have been a major contributor to the disaster through amplified accelerations. The seemingly brittle and low period URM walls therefore appear to be particularly sensitive to the peak ground acceleration. Interestingly however, very high peak ground accelerations (over 1g) generated by a small magnitude event in Eugowra (NSW) in 1994 only caused very minor damage to an URM building situated very close to the epicentre.

This apparent anomaly points to our lack of understanding in the collapse behaviour of URM walls. To implement cost-effective improvement to our URM infrastructure, the physical process leading to their collapse must be fully understood. The conventional quasi-static analysis procedure underlying current code provisions is limited to linear elastic behaviour and would not provide accurate explanations to the highly non-linear behaviour of URM walls undergoing large displacement prior to their collapses.

A simplified version of a new seismic predictive model of URM walls, developed jointly by the University of Adelaide and the University of Melbourne, is introduced in this paper to simulate the collapse behaviour of URM walls during the Newcastle earthquake. It is clearly illustrated in the model that both acceleration and displacement components of an earthquake's excitation are critical to the ultimate behaviour of URM walls and in the absence of either it is unlikely collapse will occur. Ground motion parameters used as input to the model have been derived from a separate study which is described in the companion paper entitled: "Modelling of the earthquake ground motions generated by the Newcastle earthquake".

1. INTRODUCTION

The Magnitude 5.5 earthquake which struck Newcastle, New South Wales (NSW), Australia in December 1989 was characterised by widespread damage to unreinforced masonry (URM). The most common failure mode induced by the earthquake was the transverse bending failure of URM walls. In fact more than 200 parapet and 300 simply supported wall bending failure were reported [1] being often exacerbated by soft and eroded lime mortar joints, poor workmanship and general deterioration. Damage surveillance also identified that 'Soft' soil sites could have been a major contributor to the disaster. This then lead to the common perception that the URM walls were brittle and low period and thus were particularly sensitive to the peak ground acceleration. In contradiction, a small magnitude event in Eugowra (N.S.W.) in 1994, which generated very high peak ground accelerations (over 1g), caused only very minor damage to an URM building situated very close to the epicentre. This apparent anomaly points to our lack of understanding in the collapse behavior of URM walls. To implement cost-effective improvement to our URM infrastructure, the physical process leading to their collapse must be fully understood and applied in rational analysis methodologies.

In recent years displacement based (DB) philosophies have gained in popularity for the seismic design and analysis of ductile structures although have not been thought applicable for seemingly brittle building components such as URM walls. Interestingly, extensive non-linear time history analyses (THA) and shake table testing have shown that face loaded URM walls have a significant capacity to 'rock' without failure [2]. This suggests that DB philosophies might provide a more rational means of analysis than current conventional codified quasi-static procedures. This extended abstract introduces a simplified DB analysis procedure for URM walls, developed jointly by the University of Adelaide and the University of Melbourne having been validated by non-linear THA and shake table testing [3]. The procedure is then used to simulate the collapse behavior of URM walls during the Newcastle earthquake. It is clearly illustrated in the model that both acceleration and displacement components of an earthquake's excitation are critical to the ultimate behavior of URM walls.

2. SIMULATION OF NEWCASTLE GROUND MOTION

Although the historical seismicity of Newcastle is relatively colourful with significant events recorded as far back as 1868 (ML 5.3) there was no operational seismograph in the near vicinity at the time of the 1989 earthquake. Representative ground motions for the Newcastle earthquake must therefore be simulated in order to undertake dynamic rocking analysis of URM walls.

Ground Motion Simulation for Newcastle 'Bedrock' Sites

Eighteen representative accelerograms have been generated for Newcastle 'Bedrock' sites using the computer software 'genqke' for moment magnitude M=5.6 and site distance R=15km. The companion paper 'Modelling of the Earthquake Ground Motions Generated by the Newcastle Earthquake' provides a more in depth description of generic attenuation functions used. For each of the 18 simulated accelerogram specific elastic displacement response spectrum are developed as required for displacement based analysis procedures as described in section (3).

Maximum Spectral Response Simulation for Newcastle 'Soft' Soil Sites

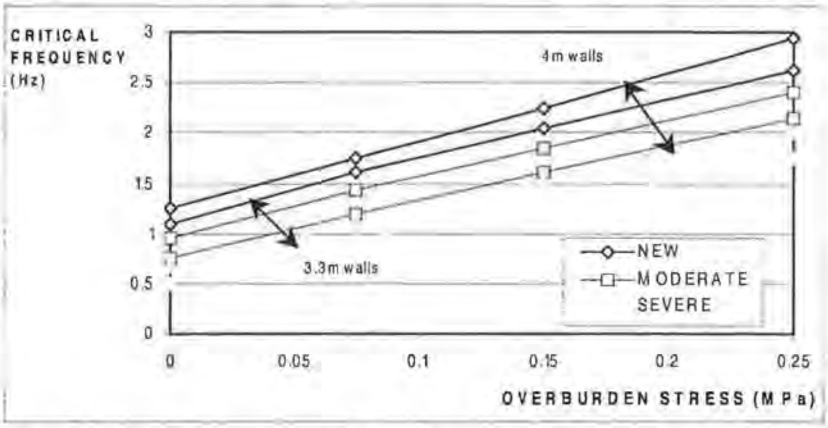
Having determined the 'Bedrock' response spectra the Frame Analogy Soil Amplification (FASA) model is applied to construct 9 Newcastle site specific response spectra that take into account the effects of resonance in the soil, refer Table (1). Further description of the 'FASA' procedure can be found in the companion paper 'Determination of Earthquake Response Spectra in Low & Moderate Seismicity Regions Using New Methodologies'. According to the 'FASA' procedure, the maximum response spectral displacement (RSD) of the soil is proportional to the predicted RSD of the underlying bedrock at the natural frequency of the site. Thus, the 'Bedrock' synthetic accelerograms generated by 'genqke' have been used as input to the 'FASA' procedure.

Table (1) – Newcastle Site Specific 'FASA' Model Results

Newcastle Site Location	Site Natural Frequency (Hz)	Peak RSD (5% Damping)	Peak RSD (3% Damping)
Newcastle Workers Club	1.21	56	67
Hamilton – James & Murray St.	1.28	57	68
Hamilton – Francis Xaviour	1.31	77	92
Newcastle RSL	2.31	43	52
Franklins (old store building)	2.13	47	56
Newcastle Technical College	1.45	56	66
Taxation Office	2.94	30	36
Tudor Inn Motel	2.74	34	40
Ambassador Hotel	1.12	83	98

3. DISPLACEMENT BASED METHODOLOGY

The displacement-based (DB) methodology is based on the comparison of displacement capacity and predicted displacement demand. To simplify complicated non-linear behaviour both displacement demand and capacity are defined using a 'substitute structure'. According to the 'substitute structure' procedure the response displacement of a SDOF system (as indicated by the RSD) can be used to represent that of a free standing parapet (cantilever) at two-thirds of the total wall height. Thus, the top of the wall displacement can be taken as 1.5xRSD. A similar scaling relationship can be used to predict the displacement of the mid-height of a wall spanning vertically between top and bottom wall supports. It can be shown that both types of URM wall described above will be safe from instability and failure provided that the wall thickness, representing displacement capacity (typically 110mm for a single leaf URM wall), exceeds 1.5xRSD representing the maximum displacement demand. If however at a given frequency the peak displacement demand exceeds the displacement capacity a more detailed analyses considering wall specific critical rocking frequency are required to determine the walls seismic behaviour and ultimate stability performance. For a rocking URM wall the critical rocking frequency can be related to the walls resonant rocking frequency at which maximum displacement amplification occurs. This in turn is related to a wall's height, overburden stress and mid-height joint degradation. Figure (1) presents critical rocking frequency for a simply supported URM wall considering the above defining parameters as determined by THA parametric studies. Importantly the critical rocking frequency increases with overburden stress, decreases with mid-height joint degradation



and wall height and is independent of wall thickness.

Figure (1) - Critical Rocking Frequency for Simply Supported URM Walls

4. ANALYSIS FOR SIMULATED NEWCASTLE GROUND MOTION

Analysed Walls

URM walls selected for analysis are typical of those found to behave satisfactorily on 'Bedrock' sites in Newcastle but observed to fail at certain 'Soft' soil locations. Walls included 3.3m and 4m tall, 110mm thick and at overburden stress levels ranging from 0MPa to 0.25MPa. Three levels of mid-height joint degradation were also considered ranging from new to severe. The reasoning behind the selection of a single leaf 110mm thick wall for analysis was that the majority of reported failures were attributed to cavity wall tie failure. This resulted in each of the two leaves effectively behaving independently generally resulting in the non-loadbearing leaf failing.

'Bedrock' Site Analysis

Intensive non-linear THA were completed for the 18 accelerograms simulated for the Newcastle 'Bedrock' site ground motion for each of the wall configurations. In agreement with observed behaviour the displacement response indicated that none of the walls would have become unstable. The dynamic analysis was then repeated using the simplified DB procedure, refer Figure (2). Here the horizontal line represents the displacement capacity of the 'substitute structure' (73mm). Below this are the 18 simulated displacement response spectrum representing the range of possible displacement demand. As for all critical wall frequency the peak RSD does not exceed the displacement capacity. The DB analysis therefore again confirms that no wall configuration would become unstable as per the observed behaviour.

'Soft' Soil Site Analysis

With reference to Table (1) the peak RSD representing the maximum displacement demand of the 'substitute structure' and the corresponding site frequency, at which the peak RSD occurs, are presented for 9 'Soft' soil sites in the Newcastle area at both 5% and 3% damping. Typically peak RSD vary from 30-100mm depending on the site natural frequency that in turn depends on the depth of the soil. Thus, for some sites the peak RSD exceeds the displacement capacity (73mm) indicating instability although for other sites the capacity exceeds demand indicating stability. Therefore, as was observed in Newcastle, certain wall configurations would be expected to fail on some 'Soft' soil sites and not others. For 'Soft' soil sites with RSD greater than 73mm the corresponding site frequency ranges from 1.12Hz to 1.31Hz. Again referring to Figure (1) we can see that walls with critical frequencies in this range and as such which would respond to the peak RSD are non-loadbearing (OMPa overburden stress) walls. Therefore it can be concluded that the analysed single leaf 110mm non-loadbearing walls would have been expected to fail during the Newcastle earthquake at certain 'Soft' soil sites but not others. This is again in agreement with observed failures.

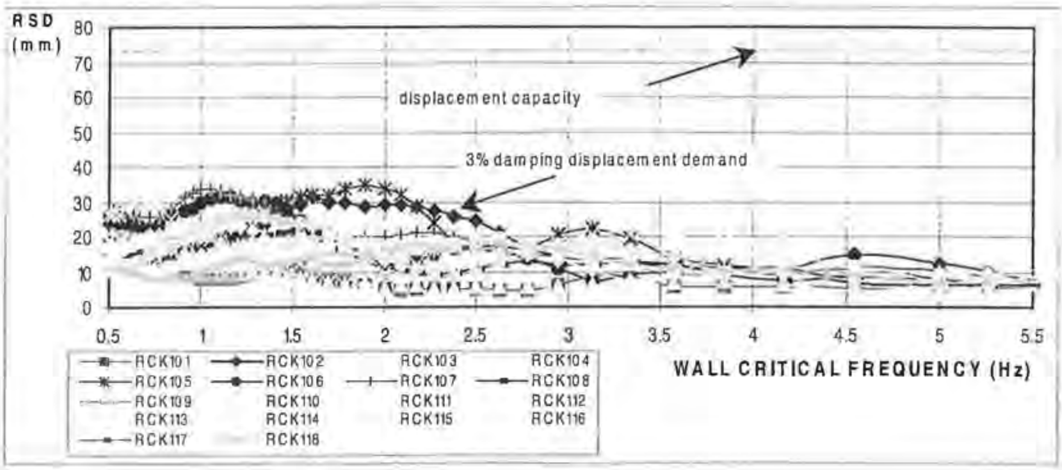


Figure (2) 'Bedrock' Site DB Analysis

5. SUMMARY

A simplified DB analysis procedure has been presented for face loaded URM walls validated by extensive THA and shaking table tests. Simulated Newcastle earthquake ground motions for both 'Bedrock' and 'Soft' soil sites were then used to analyse typical URM walls with good correlation found between analytical and observed failures. This study reinforces the effectiveness of the DB methodologies for the analysis of face loaded URM walls. Further, it dispels the common misunderstanding that stability of URM walls is solely dependant on the instantaneous acceleration or inertia force which is assumed in traditional codified quasi-static analysis procedures but rather on a combination of both ground acceleration and displacement.

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